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Structural performance of unprotected concrete-filled steel hollow sections in fire: A review and meta-analysis of available test data

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Abstract. Concrete filled steel hollow structural sections (CFSs) are an efficient, sustainable, and attractive option for both ambient temperature and fire resistance design of columns in multi-storey buildings and are becoming increasingly common in modern construction practice around the world. Whilst the design of these sections at ambient temperatures is reasonably well understood, and models to predict the strength and failure modes of these elements at ambient temperatures correlate well with observations from tests, this appears not to be true in the case of fire resistant design. This paper reviews available data from furnace tests on CFS columns and assesses the statistical confidence in available fire resistance design models/approaches used in North America and Europe. This is done using a meta-analysis comparing the available experimental data from large-scale standard fire tests performed around the world against fire resistance predictions from design codes. It is shown that available design approaches carry a very large uncertainty of prediction, suggesting that they fail to properly account for fundamental aspects of the underlying thermal response and/or structural mechanics during fire. Current North American fire resistance design approaches for CFS columns are shown to be considerably less conservative, on average, than those used in Europe.

Keywords: concrete filled steel sections; fire resistance; standard fire testing; structural design.

1. Introduction

Concrete filled steel hollow structural (CFS) sections are an architecturally, economically, and environmentally attractive means by which to support large compressive loads in buildings. They consist of hollow steel tubes that are in-filled with concrete to provide optimized load carrying capacity and structural fire resistance as compared with unfilled steel tubes. The concrete infill and the steel tube work together to yield several key benefits, both at ambient temperature and during a fire. The steel tube acts as stay-in-place formwork during casting of the concrete, thus reducing forming and stripping costs, it laterally confines the infill concrete which can enhance the concrete's compressive strength and axial deformability, and it provides a smooth, rugged, architectural surface finish. The concrete infill drastically enhances the fire performance of the column by allowing the heated steel tube to shed a

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portion of the axial load demand to the concrete core when heated during a fire, in some cases providing adequate fire resistance without any applied fire protection to the tube. The infill concrete also enhances the steel tube's resistance to elastic local buckling (Kodur 2007).

Various types of concrete filling may be used in practice, including unreinforced concrete (PC), steel bar reinforced concrete (RC), and steel fibre reinforced concrete (FIB). A current trend in ambient temperature structural design of CFS columns is for the infill concrete to have a high-compressive strength of more than 60 MPa (up to 100 MPa in some cases). These members are now widely specified by architects and engineers and are increasingly being applied in the design and construction of multi-storey and high rise buildings within highly optimized structural frames, where structural fire resistance ratings of two hours or more may be required (Communities and Local Government 2007).

Fire resistance design guidance is available (Kodur 2007, Lennon *et al.* 2007, CEN 2005, CEN 2008, Wang and Orton 2008, Aribert *et al.* 2008, Park *et al.* 2008) for most types of CFS columns. However, much of the available guidance was developed for conventional applications based on large-scale standard furnace tests and computer modelling of short, concentrically loaded, small-diameter columns envisioned for use in braced frames using normal strength (55 MPa or less) concrete. Current fire resistance design approaches are therefore rather limited in scope, and this makes the design of CFS columns using performance-based approaches difficult to defend to approving authorities.

Fire tests performed to date have generally used ASTM-E119 (ASTM 2007) or ISO-834 (ISO 1975) time temperature curves. Neither of these fire scenarios represents well a real fire in a modern, open plan, multi-storey building (Stern-Gottfried *et al.* 2010, Law *et al.* 2011) nor parametric (natural) design fires (CEN 2009) which are increasingly being used in fire engineered building designs. The materials of construction specified in modern designs are different in a host of ways from those that were considered in older tests, mostly concerning the infill concrete. For instance, in practice polypropylene and steel fibre blends are being added to infill concretes with strengths in excess of 60 MPa to discourage spalling and to allow removal of internal steel reinforcing cages. Many thermal and structural response phenomena occur during heating of a CFS column that influence failure but which cannot currently be predicted with confidence (Wang and Orton 2008). The appropriate effective length of CFS columns in fire has received considerable research attention (Wang 2005), yet available guidance, based almost entirely on computational modelling, may be unconservative. For instance, the location of local buckling and failure of a CFS column in fire remains difficult (or impossible) to predict (Wang 2005).

The meta-analysis in this paper seeks to understand if some of the above uncertainties play a role in the predictive performance of available fire resistance design approaches. The paper provides a review of available research on the fire performance of CFS columns covering experimental testing and computational modelling carried out worldwide, and compares available code-based fire resistance design approaches for CFS columns in North America (Kodur 2007) and Europe (CEN 2005). It focuses on the assumptions made in, and the uncertainties associated with, available research (whether experimental or numerical), with a view to highlighting the uncertainties of available design approaches. The overarching goal is to support performance-based design and application of CFS columns in multi-storey buildings.

2. CFS columns in fire

It is worth briefly reviewing the 'typical' response of a CFS column in fire (or rather, in a standard

furnace test) to frame the discussions which follow. The four typical stages of deformation of a concentrically loaded CFS column in fire are well documented in the literature (Wang and Orton 2008). In Stage I, as the steel tube heats up it expands both in the horizontal (lateral) and vertical (longitudinal) directions. The steel, having a higher coefficient of thermal expansion and heating more rapidly, expands at a faster rate than the concrete infill and this creates a gap between the steel tube and the concrete and allows the steel to expand unrestrained. The consequences of this expansion in the lateral direction are not well known, although it appears that there are both thermal effects (i.e. reduction of heat transfer to the concrete and effective insulation of the back face of the steel tube) and structural effects (i.e. removal of local buckling restraint of the tube wall). In the axial direction, thermal expansion of the tube causes it to carry more of the axial load as it expands longitudinally. This longitudinal expansion continues until the tube takes so much of the load that it yields locally in compression and rapidly shortens and sheds load back to the concrete core (Stage II). Provided that the column remains stable during this contraction (which is *not* assured on the basis of the available data) the load will continue to be carried by the concrete core with only minor changes in column length as the fire continues (Stage III). The core continues to carry the load until there is sufficient degradation of the concrete so that the load can no longer be supported and the column fails (Stage IV). It should be noted that a stable Stage III is the most important factor for achieving large fire resistances in CFS columns, but that for reasons unknown this stage is sometimes absent (Wang and Orton 2008).

3. Furnace test data

More than 300 large scale standard fire tests have been carried out globally on CFS columns of various types. A comprehensive summary of the available test data and relevant parameters is given in Table 1-Table 4; these divide the available data into tests on concentrically loaded unprotected columns (Table 1), eccentrically loaded unprotected columns (Table 2), concentrically loaded protected columns (Table 3), and eccentrically loaded protected columns (Table 4). The thermal exposure in all cases was based on a *standard* fire similar or identical to the ISO 834 fire (ISO 1975). The main contributors to the available test database for concrete-filled SHS are the National Research Council of Canada (NRCC) and the Comité International pour le Développement et l'Etude de la Construction Tubulaire (CIDECT).

3.1. Steel tube characteristics

The dimensions and strength of the steel tube section as compared with the concrete infill play central roles in the fire performance of CFS columns. These parameters dictate the relative contributions of the steel tube and the infill concrete to the overall load carrying capacity of the column, both at ambient temperature and during fire. In general, unprotected columns, which rely more heavily on the steel tube, will tend to be more critical in fire since they lose a greater proportion of their strength due to heating of the external steel tube. However, thinner walled tubes are more likely to buckle locally and this may affect both the effective length of a column during fire and its axial crushing strength. Interestingly, it appears that the specific factors leading to and the consequences of local buckling of the steel tube on the fire performance of CFS columns have received only limited research attention (Ding and Wang 2008).

Furnace tests performed to date have considered steel tube thicknesses ranging from 3.6-16 mm, with

tube-to-infill cross-sectional area ratios as low as 0.9% and as high as 5.1%. Data from tests on circular sections from 121-600 mm in diameter are available in the literature, and on square sections ranging from 100-350 mm in minimum side length. However, the vast majority of tests ($\approx 85\%$) have been on columns with a largest minimum dimension of less than 300 mm, and only a single test has ever been performed on a column over 478 mm in largest minimum dimension; in this case the load ratio (i.e., the ratio of the applied forces in fire conditions to the design load capacity of the member at room temperature) was unrealistically low (about 0.2). Despite the obvious practical difficulties in performing realistic fire tests on members larger than 600 mm in diameter, the lack of fire test data for very large columns is currently a limiting factor in applying design procedures (Kodur 2007). It would seem reasonable, however, to assume that once the fundamental mechanics of CFS columns are understood and appropriately modelled during fire that larger sections could be designed using validated analysis tools. Steel strengths represented in the available fire test data range from 240 to 510 MPa.

The cross-sectional shape (circular or square) of the column plays important roles in the response of CFS columns during fire. At ambient temperatures circular tubes uniformly confine the concrete core as axial loads are increased, so that the concrete is placed in a state of triaxial stress which increases both its strength and its axial-flexural deformability; square or rectangular columns provide non-uniform confinement with only minimal increases in strength but considerable enhancements in deformability. Loss of confinement due to excessive heating of the steel tube during a fire, which in addition to reductions in the strength and stiffness of the tube may also cause separation from the core due to differential thermal expansion, will result in a greater proportional loss of strength for a circular column than for a rectangular one. Furthermore, circular sections exposed to uniform heating will heat up uniformly, whereas square sections will heat more rapidly at the corners, potentially inducing additional thermal stresses within the cross-section. The potential influence of cross-sectional shape on the issues noted above, the observed failure modes, and the performance of applied fire protection have received little attention.

3.2. Concrete infill material

The type of concrete infill within a CFS column (PC, RC, or FIB) drastically affects its fire performance. Unprotected PC filled CFS columns fail at comparatively low loads when exposed to fire. Rapid loss of strength and stiffness of the fire exposed steel tube cause loads to be shed to the concrete and, depending on the level of axial and flexural loads in the section, can lead to excessive local stresses in the concrete which cause failure (Kodur 2007). PC infill has been used in the majority of tests available in the literature ($\approx 68\%$). RC and FIB infill have been used in 25% and 7% of tests, respectively.

79% of available tests have used concrete with $f_c' < 50$ MPa and only 8% have used $f_c' > 70$ MPa. This tendency towards lower concrete strengths reflects the fact that the bulk of the tests ($\approx 63\%$) were performed prior to 1980, so that the tested concrete strengths were representative of those being used at that time. These are not reflective of current practice as concrete specified in CFS columns in multi-storey building designs now tends toward 70 MPa or higher. Studies focused specifically on the response to fire of CFS columns with f_c' up to 100 MPa have more recently appeared in the literature (Lu *et al.* 2009, Han *et al.* 2003, Hass *et al.* 2001, Kodur and Latour 2005).

The introduction of internal steel reinforcement within the concrete core increases the fire resistance of a CFS column, particularly under axial-flexural loading. In addition to carrying a portion of the total

loads on the column once the steel tube is heated, the internal steel reinforcement also acts to decrease the propagation and localization of cracks within the concrete and slows the loss of strength on further heating (Myllymaki *et al.* 1994). The increase in fire resistance depends on many factors, however the reinforcement ratio and depth of concrete cover dominate.

Core steel reinforcement ratios between 1.0-5.1% have been tested, although the vast majority (> 80%) have used between 1.0-3.0%, with between four and eight longitudinal bars with square ties or steel spirals. These are comparable to reinforcement ratios which would typically be found in conventional concrete columns. Reinforcement ratios above 3.0% provide comparatively little improvement in fire resistance (Stanke 1975).

While RC filled SHS columns perform well in fire and can typically be designed without any need for applied fire protection, there are many practical concerns associated with the placement of the internal steel cages, which can be costly and time consuming. Thus, RC infill is unfavoured in modern CFS column designs. There is a clear trend toward the use of PC infill which has better constructability but considerably reduces fire resistance and can force the use of applied fire protection (bringing considerable additional costs and construction issues). One possible means to avoid having to use applied fire protection, first examined in the late 1970s with limited success (CIDECT 1976) but more recently with increased success (Kodur and Lie 1995), is to use FIB infill. The advantages of FIB over PC infill is that suitably proportioned steel fibres within the concrete arrest the propagation of micro cracks and improve the continuity of the concrete core and its ability to carry load. Furthermore, the fibres enhance the tensile strength of the infill concrete, potentially allowing it to carry tensile/flexural loads during fire (yet to be experimentally confirmed). FIB infill can provide fire resistance values which are comparable to those of RC filled SHS columns (Kodur and Lie 1996a), although this has only been shown for concentric loading for one steel fibre type and volume content (1.8% fibres by mass). The mechanics of FIB infill CFS columns in fire remain poorly understood.

The type of aggregate used in the infill concrete also plays a role since different aggregates may result in an order of magnitude difference in the coefficient of thermal expansion of the core. This may impact heat transfer within the section and the formation and size of the air gap. It may also affect the transfer of load from the steel tube to the concrete core, and hence the column's deformation and ultimate failure mode. Only limited research has considered the possible effect(s) of aggregate type on CFS columns. For instance, an NRCC study (Chabot and Lie 1992) showed that a siliceous aggregate RC filled CFS column tested at a load ratio of 0.58 had half the fire endurance of an equivalent CFS column with carbonate aggregate infill tested at the same load ratio.

3.3. Slenderness & rotational restraint

A column's relative slenderness and end fixity is crucial when considering its response to fire, particularly in terms of the observed failure mode(s) since slender columns are more likely to fail by *global* buckling whereas short columns will fail by *local* buckling and/or crushing of the core. Column lengths between 760-5800 mm are represented in the literature, although the vast majority ($\approx 83\%$) are between 3030 and 3810 mm. This is due to the size of available standard testing facilities globally. The lack of data from realistic fire tests of slender CFS columns is currently claimed to limit their application (Kodur 2007). However, the most slender CFS columns reported in the literature have non-dimensional slenderness (calculated according to Eurocode procedures (CEN 2008, CEN 2005, Lennon *et al.* 2007)) of about 1.3. This slenderness is well within the practical range for CFS columns that are likely to be considered in all but a tiny fraction of practical design situations.

Several column end fixity combinations are represented in the literature and are given as: fixed-fixed (FF), pinned-fixed (PF), and pinned-pinned (PP). The majority of tests ($\approx 64\%$) have been on FF members, although it is worth noting that the true fixity during testing is never perfect and is usually not known. It is also noteworthy that columns are rarely heated over their entire height during furnace testing. For instance, in the NRCC testing furnace columns are heated over only 80% of their total length (Chabot and Lie 1992, Kodur and Lie 1996a, Kodur and Lie 1995) with the ends insulated. This has potentially important implications, particularly for FF and PF columns when relating the non-dimensional slenderness at ambient conditions to the effective slenderness of the column during a fire test. Unheated regions will maintain their full flexural stiffness during a fire test, which artificially reduces the assumed effective slenderness during the test as compared with a column in a real building. Furthermore, there is compelling evidence from non-standard furnace tests performed on CFS columns, which included load introduction regions with beams framing into the columns during the tests, that end fixity, load introduction, axial load ratio, and steel tube thickness all influence both the likelihood and location of local buckling of the steel tube during fire – the location of the local buckle being the primary factor dictating the effective length of the column during a fire (Wang and Davies 2003). Thus, FF and FP furnace tests may be unconservative with respect to the true effective length of CFS columns during fire, unless the true end fixities of the tested columns are accounted for in considerable detail. This issue is particularly noteworthy given that Eurocode (CEN 2005) provisions permit the effective length factors of columns in non-sway frames to be taken as 0.5 during fire if the columns are continuous across multiple floors, whereas test data suggest that local buckling of the steel tube may lead (unconservatively) to an effective length of 1.0 times the storey height regardless of the end fixity condition or continuity (Wang and Davies 2003).

3.4. Load eccentricity & bending

The relative importance of load eccentricity and bending depends predominantly on the type of concrete infill. The majority of available tests ($> 80\%$) have been on CFS columns under concentric load. Intentionally applied load eccentricity ratios between 2.5% and 150% are present in the literature. However, the most rational way to test the specific impacts of load eccentricity for various types of concrete infill would be to test identical CFS columns with different initial load eccentricities but at the same load ratio; such comparative data are scarce.

The available data clearly show that CFS columns filled with PC are highly sensitive to load eccentricity, and that they suffer large reductions in fire resistance times under loads of increasing eccentricity (all other factors being equal). This is due to the fact that PC infill is severely limited in its ability to carry flexural loads once the steel tube heats and sheds its load to the concrete core. Unprotected CFS columns with PC infill are generally not used where load eccentricity or bending (including slenderness effects) are expected during a fire. When identical CFS columns with PC infill are tested under different initial load eccentricities but at the same fire test load ratio, the specific impact of eccentricity is less severe, although the available data are highly contradictory. For unprotected PC filled CFS columns with identical load ratios the available data (Han *et al.* 2003, Lu *et al.* 2009) suggest that initial eccentricity ratios as high as 30% may have no obvious detrimental effect on fire resistance (albeit with fire resistances of less than 30 min in all cases). For fire protected PC filled CFS columns however, data (Sakumoto *et al.* 1994) show that eccentricity ratios of only 10% may cause reductions in fire resistance of up to 40% (with a fire resistance of 166–188 mins for concentric loading). For unprotected CFS columns with RC infill, no negative influence of eccentricity or bending is expected

within the practical range of internal steel reinforcement ratios for columns with the same load ratio (Kordina and Klingsch 1983). While research has suggested that use of FIB infill can improve the fire performance of concentrically loaded CFS columns with unreinforced concrete infill, the benefits of FIB infill for columns with eccentric loads and/or bending have not been properly investigated.

3.5. Load ratio

In practice, the load ratio for a structural member typically lies somewhere in the range of 0.3 to 0.5 (Buchanan 2002), and in some cases up to 0.6. The fire resistance of any type of column is explicitly linked to the sustained load applied during testing, with higher load ratios leading to lower fire resistance. Load ratios of less than 0.3 ($\approx 30\%$ of tests) are likely to be unrealistically low, and greater than 0.6 ($\approx 21\%$ of tests) are unrealistically high.

3.6. Failure modes

It is unfortunate that available testing reports from fire tests on CFS columns devote relatively little attention to describing the observed failure modes in any significant detail, since this information is of fundamental importance in understanding the mechanics at play during fire. Tests have generally been grouped into two broad categories: buckling and crushing. Global buckling failures occur when three locations of little to no rotational restraint (hinges) develop a collapse mechanism in a column and large lateral deflection of the column occurs. Hinge locations are associated with areas of local buckling of the steel tube, yet the factors influencing the formation of these hinge regions (including load introduction, rotational restraint, inter-storey effects, localized heating) remain poorly understood (Wang and Davies 2003). As expected, buckling failures are prevalent in slender columns with eccentric loads. Crushing failures occur where degradation of the core concrete's strength is sufficient that the load can no longer be supported. Such failures are typically accompanied by local 'elephant's foot' buckling of the steel tube; this has been observed coincident with the majority of crushing failures, however the location of the local buckles varies from test to test. Of the available tests for which failure modes are clearly quoted (96 tests), 47% are stated as 'buckling' and 53% as 'crushing'. Very importantly, it has been observed that not all CFS columns are able to transition from Stage II to Stage III; some fail shortly after first yielding of the steel tube. Wang and Orton (2008), note that whether a CFS column is able to pass through all four stages of deformation depends on many factors, although risk factors include slenderness, low internal reinforcement ratio, high applied load, load eccentricity, and (for reasons unknown) stiff rotational restraint 'at the top'. There is currently no simple method to identify CFS columns that are not able to go through all four stages; evidence of a fundamentally incomplete understanding of the mechanics and interactions leading to failure.

4. Computational modelling

Many approaches have been used to predict the fire resistance of CFS columns. A suitably validated model can be used to perform parametric studies on various column parameters and develop simple analytical formulae and procedures for column design, without the need to test large numbers of specimens. Validated 3D finite element (FE) models can also be used to study the specific impacts of key issues (e.g., non-standard heating regimes, air gap formation, local buckling, longitudinal slip

Table 1. Unprotected Concentrically Loaded CFS Columns

Researchers		Specimen details				Steel Tube				Concrete					Failure							
Name ^{Ref}	Date	Length	End Conditions		Load Ratio	Section Shape		Section Size	Wall Thickness	Steel Strength	Concrete Type		Strength		A_s/A_c Ratio	Cover	Time		Mode			
		<i>m</i>						<i>mm</i>	<i>mm</i>	<i>MPa</i>			28 - Day <i>MPa</i>	Test <i>MPa</i>		<i>mm</i>	C <i>min</i>	S <i>min</i>	LB	GB C		
NRCC ^a	1982-95	3.81	PP	5	<0.3	40	C	51	141 - 406	4.8 - 12.7	300 - 350	PC	45	24 - 91	12 - 107	-	-	48-294	62-131	24	21	
			PF	-	<0.6	28	S	22	150 - 305	5 - 12.7	300 - 419	RC	12	38 - 82	38 - 93	2.1 - 2.5%	40	43-188	39-212	6	6	
			FF	68	>0.6	4						FIB	16	41 - 90	39 - 100	1.77%	-	65-259	60-128	7	9	
CIDECT ^{# b}	1954-76	3.6 to 4.8	PP	7	<0.3	7	C	17	121 - 600	3.6 - 16	240-420	PC	16	21-46	30-52	-	-	36	24-165			
			PF	-	<0.6	32	S	35	140 - 300	3.6 - 8	300-429	RC	33	7-95	7-95	1.3-3.7%	25-50	12-198	16-192	??		
			FF	45	>0.6	5						FIB	3	47-52	50-56	4.30%	-	-	19-80			
CIDECT ^c	1977-2000	0.8 to 5.8	PP	7	<0.3	34	C	19	159 - 324	3.6 - 8	286-410	PC	38	31-96	34-96	-	-	28-102	15-134			
			PF	4	<0.6	20	S	33	150 - 350	4 - 10	243-550	RC	12	32-99	38-99	1 - 2.9%	35-43	65-134	51-135	??		
			FF	41	>0.6	-						FIB	4	40-98	48-98	2.8 - 4.3%	-	77	55-81			
Han ^d	2003	3.81	PP	6	>0.6	6	C	4	150-478	4.6-8	259-381	PC	6	40-69	49	-	-	20-29	16-21	1	4	1
							S	2	150-200	8	341											
Suzuki ^e	1990	1.32 – 2.99	PP	9	<0.3	9	S	16	200-300	4.5-9	293-349	PC	13	28-30	28-32.6	-	-	-	76-152	??		
			PF	7	<0.6	7						RC	3	30	32.6	1.53%	17.5	-	78-147			
Sakumoto ^f	1993	3.5	PP	1	<0.3	1	S	1	300	9	358	PC	1	??	37.5	-	-	-	33	1*	1*	
Kim ^g	2005	3.5	PP	20	<0.6	20	C	10	319-406	7-9	304-311	PC	20	28 - 38	??	-	-	28-150	44-160	??		
							S	10	300-350	9	363											
Lu ^h	1993	0.76	FF	4	<0.3 >0.6	2 2	S	4	150-200	5-6	467-486	PC	4	90-99	??	-	-	-	26-92	4*	4*	

Table 2. Unprotected Eccentrically Loaded CFS Columns

Researchers		Specimen details			Steel Tube				Concrete					Failure								
Name ^{Ref}	Date	Length	End Conditions	Load Ratio	Section Shape	Section Size	Wall Thickness	Steel Strength	Concrete Type	Strength		A_s/A_c Ratio	Cover	Time		Mode						
		<i>m</i>				<i>mm</i>	<i>mm</i>	<i>MPa</i>		28 - Day <i>MPa</i>	Test <i>MPa</i>		<i>mm</i>	C <i>min</i>	S <i>min</i>	LB	GB C					
NRCC ⁱ	1990-94	3.81	PP	3	<0.3	2	C	1	219	8.2	350	PC	1	24.3	31.9	-	-	33	-	-	1	-
					<0.6	1	S	2	300	8	394	RC	2	40.7	43.8	5.07%	40	-	58-126	-	2	-
CIDECT ^{#j}	1977-82 & 2001	Eccentricity ratio			15-40%																	
		3.03	PP	2	<0.3	2	C	5	133-356	4-6	235-383	PC	10	30-64	30-64	-	-	33-69	22-112			
		to	PF	37	<0.6	24	S	35	100-300	4-12.5	234-550	RC	30	27-75	27-75	0.9-4.4%	15-43	45-56	23-92	??		
		5.2	FF	1	>0.6	6																
Han ^k	2003	Eccentricity ratio			2.5-150%																	
		3.81	PP	6	>0.6	6	C	4	219-478	4.6-8	293-381	PC	6	40-69	49	-	-	7-32	20-24	-	4	2
					S	2	150-200	8	341													
Lu ^l	2003	0.76	FF	2	<0.6	2	S	2	150-200	467-486	394	PC	2	90-99	??	-	-	-	43-55	2*	-	2*
		Eccentricity ratio			12.5-17%																	

Tables 1 & 2, notations : ϕ - rectangular (300 x ϕ); * local buckling occurred first followed by crushing of the concrete; # Specimen details not fully known

Table 3. Protected Concentrically Loaded CFS Columns

Researchers		Specimen details				Steel Tube				Concrete					Failure					
Name ^{Ref}	Date	Length	End Conditions		Load Ratio	Section Shape	Section Size	Wall Thickness	Steel Strength	Concrete Type		Strength		A _s /A _c Ratio	Cover	Time		Mode		
		<i>m</i>					<i>mm</i>	<i>mm</i>	<i>MPa</i>			28 - Day <i>MPa</i>	Test <i>MPa</i>		<i>mm</i>	C <i>min</i>	S <i>min</i>	LB	GB C	
CIDECT ^{m#}	1971-75	3.6	PP	2	<0.3	3	C 5	168-219	3.6-12.5	300-360	PC 33	18-51	??	-	-	-	46-290			
			FF	43	<0.6	12	S 40	140-330	3.6-8	355-388	RC 12	45-47	41 - 55	1.1-3.7%	25-33	64-90	35-130		??	
					>0.6	30														
Intumescent Paint - 15 (1 - 2 kg/m ²), 6 - Rock Wool, 2 - Liquid Stone [¥] , 9 - Vermiculite Boards, 4 - Plaster, 3 - Plaster Shells, 2 - Alphapan [§] , 4 - Asbestos Cement																				
Suzuki ⁿ	1990	2.99	PP	4	<0.3	2	S 4	200-300	6-9	313-327	PC	4	28	28	-	-	-	120 [§]	??	
					<0.6	2														
Rock Wool (30mm thick) – 4,																				
Sakumoto ^o	1993	3.5	PP	4	<0.3	4	S 4	300	9	358-361	PC	4	??	38	-	-	-	166-194	4*	4*
					>0.6	1														
Ceramic Board – 3 Intumescent Paint -1 (1.25 kg/m ²)																				
Han ^p	2003	3.81	PP	11	>0.6	11	C 5	150-478	4.6-8	259-381	PC	11	18-69	19-49	-	-	120-196	78-169	10	1
					>0.6	1	S 6	150-350	5.3-8	246-341										
Intumescent Paint - 11 (4.4 - 10 kg/m ²)																				
Edwards ^q	1997	3.6	FF	6	<0.6	5	C 2	168-324	6.3	306-321	PC	6	34-43	43-48	-	-	115-166	102-146		??
					>0.6	1	S 4	150-300	6.3-16	331-375										
Intumescent Paint - 6 (0.8 - 1.1 kg/m ²)																				

¢ - 4 rectangular, 2 square * local buckling occurred first followed by crushing of the concrete # Specimen details not fully know

¥ - Liquid Stone protection consisted of vermiculite particles mixed with a synthetic stone produced by a reaction of calcite and portlandite

§ - Alphapan is a form of protection consisting of panels cut from plates made of agglomerated rock fibres

Table 4. Protected Eccentrically Loaded CFS Columns

Researchers		Specimen details			Steel Tube				Concrete					Failure					
Name ^{Ref}	Date	Length	End Conditions		Load Ratio	Section Shape	Section Size	Wall Thickness	Steel Strength	Concrete Type	Strength		A_s/A_c Ratio	Cover	Time		Mode		
		<i>m</i>					<i>mm</i>	<i>mm</i>	<i>MPa</i>		28 - Day <i>MPa</i>	Test <i>MPa</i>		<i>mm</i>	C <i>min</i>	S <i>min</i>	LB	GB	C
Sakumoto ^r	1993	3.5	PP	4	$\frac{<0.3}{<0.6}$ $\frac{3}{1}$	S 4	300	9	358-361	PC 4	??	38	-	-	-	88-148	-	4	-
		Ceramic Board – 3			Intumescent Paint -I (1.25 kg/m ²)					Eccentricity ratio		10%							
Han ^s	2003	3.81	PP	1	$\frac{>0.6}{>0.6}$ $\frac{1}{1}$	S 1	350	8	284	PC 1	18	19	-	-	-	108	-	-	1
		Intumescent Paint -I (2.8 kg/m ²)					Eccentricity ratio		30%										

Abbreviations used in tables: PP – Pinned-pinned, PF – Pinned-fixed, FF – Fixed-fixed, C – Circular, S – Square, PC – Plain concrete, RC – Reinforced concrete, FIB – Fibre reinforced concrete, As – Cross-sectional area of steel, Ac – Cross-sectional area of concrete, LB – Local buckling, GB – Global buckling, C – Crushing.

Table 1 references: a) Kodur and Latour 2005; Myllymaki et al. 1994; Kodur and Lie 1995; Chabot and Lie 1992; Kodur and Lie 1996b; Lie and Chabot 1992; b) Stanke 1975; CIDECT 1976; Grandjean et al. 1981a; Grandjean et al. 1981b; c) Hass et al. 2001; CIDECT 1976; d) Han et al. 2003; Han et al. 2003; e) Suzuki et al. 1985; Kimura et al. 1990 f) Sakumoto et al. 1994; g) DK Kim et al. 2005; h) Lu et al. 2009. **Table 2 references:** i) Myllymaki et al. 1994; Lie and Chabot 1992; j) CIDECT 1976; Kordina and Klingsch 1983; Klingsch and Wittbecker 1988; Renaud and Joyeux 2001 ; k) Han et al. 2003; Han, Yang, and Xu 2003; l) Lu, Zhao, and Han 2009. **Table 3 references:** m) CIDECT 1976; n) Suzuki et al. 1985 o) Sakumoto et al. 1994; p) Han et al. 2003; Han, Yang, and Xu 2003; q) Edwards 2000. **Table 4 references:** r) Sakumoto et al. 1994; s) Han et al. 2003; Han et al. 2003

between the concrete and the steel tube) which cannot be easily captured using simple models. However, all of the computational modelling approaches available in the literature depend on user inputs for a wide variety of parameters for which limited guidance is available (e.g. emissivity is taken anywhere between 0.5-1.0, often with little justification) and have been validated by comparison against ‘selected’ test data.

4.1. Simple crushing analyses

The simplest models presented in the literature predict only the cross-sectional crushing strength of CFS columns (Chung *et al.* 2008, Yin *et al.* 2006). These models apply stress-strain curves for the columns’ constituent materials at elevated temperature and assume that: the thermal and structural behaviour of the member is uncoupled, there is perfect bond between the steel tube and the infill, no gaps form between the tube and the concrete, no slip occurs between the tube and the concrete, and neither local nor global buckling need be considered; all of these assumptions are known to be false, but the degree to which they influence the models’ predictive abilities is not yet known. Such an approach is potentially appropriate only for stocky columns; based on the most-observed failure modes in experiments, such approaches appear indefensible in most cases. Indeed, in a study by Chung *et al.* (2008) the crushing model consistently over-predicted both temperatures and failure times.

4.2. Cross-sectional equilibrium approaches

Several models in the literature have taken an approach based on a cross-sectional equilibrium analysis (Kodur and Lie 1996b, Han *et al.* 2003). The column’s cross-section is divided into annular or square elements and sectional equilibrium at mid height is used through an iterative analysis to develop curves of capacity versus time of fire exposure. These models assume that the concrete has no tensile strength, plane sections remain plane, there is perfect bond between steel and concrete (and thus no slip, air gap, or local buckling), there is no composite action between the steel and the concrete, and no concrete confinement due to the steel tube. It is further assumed that effective length of the fixed-fixed column remains uniform throughout the heating at $0.7L_{cr}$ (*chosen* to match test data from the NRCC column furnace). The deflected shape of the column is assumed as sinusoidal – therefore prescribing the failure mechanism for the column as one with a single hinge at mid-height. Comparison against results from *selected* NRCC tests has shown this approach to conservatively predict fire resistance.

4.3. Custom finite element packages

Several custom FE packages, developed specifically for structural fire analysis and incorporating varying degrees of complexity, have been applied to CFS columns. This includes independent work by Schaumann *et al.* (2009), Kodur and Fike (2009), and Renaud (2004). These analyses all differ in many subtle respects which are not important for the current discussion. What *is* important is that neither Schaumann *et al.* (2009) nor Kodur and Fike (2009) apparently included the effects of gap formation, slip between the SHS tube and the concrete, confinement, or local buckling, despite the fact that the authors themselves highlighted their potential importance. Renaud’s (2004) comprehensive analysis *does* consider the thermal impacts of gap formation (albeit by *imposing* a predefined thermal resistance so as to match test observations) as well as the structural impacts of slip between the steel tube and the infill concrete (using a special connection element), although it appears not to consider local buckling

of the steel tube. Renaud's analysis is validated against 33 tests (yet 300+ tests are available); it is not clear why these specific tests were chosen. A notable conclusion of Renaud's study was that slip appears to play an important role, particularly within the first 30 minutes of a standard fire test with PC infill or for columns with eccentric load, bending, or high slenderness.

4.4. General purpose finite element models

Several studies have used general purpose FE packages to perform structural fire analyses of CFS columns. Zha (2003) presents a 3D FE model of a circular CFS column exposed to a standard fire using DYNA3D, apparently neglecting gap formation, slip, concrete confinement, and local buckling. This is validated *only* against the highly conservative tabular design approach given in Eurocode 4 (CEN 2005) rather than against real experimental data. Hong and Varma (2009) used ABAQUS to model the standard fire behaviour of CFS columns and, while ignoring the influence of gap formation, included slip and local buckling in their analysis by *manually* de-bonding the steel tube over a prescribed length near the column mid height. The model was validated against 15 tests selected from the literature; again it is unclear why these specific tests were chosen. This study confirmed that the effects of local buckling and slip are more important for columns which experience bending. Espinós *et al.* (2009) also used ABAQUS, neglecting gap formation, confinement, slip, and local buckling, and verified their model against only eight experimental results, none of which had load ratios above 0.3.

The most advanced 3D FE modelling presented to date is by Ding and Wang (2009) using ANSYS. This study considered the potential thermal and structural impacts of gap formation and slip between the steel tube and the infill concrete, as well as local buckling of the steel tube. The thermal influence of an air gap was modelled by assuming a constant air gap of 1 mm with an assumed associated thermal resistance *chosen* to match selected tests available in the literature. The resulting thermal analysis indicated that the accuracy of temperature prediction in CFS columns in fire can be noticeably improved by accounting for the formation of an air gap. Given the number of parameters upon which the formation of an air gap depends, research is needed to understand and model this process for the range of steel sections and concrete infill materials currently used in practice. Slip was considered using 3D surface-to-surface contact elements and a Coulomb friction model. The results of parametric studies to investigate the potential effects of slip on lateral deflection response and time to failure indicated that the effects were minor. On the basis of their work, Ding and Wang (2009) concluded that it was not absolutely essential to include slip in the analysis, although slightly better results were obtained when slip was included, that the specific properties of the bond-slip response were of little significance as long as slip was included, and that introducing an air gap improved the accuracy of the thermal analysis and hence the structural performance predictions.

5. Knowledge Gaps

5.1. Fire scenario

Current fire design procedures for CFS columns are based on furnace testing using standard fires. This is clearly inaccurate to model real fires (Beyler *et al.* 2007), and advanced structural fire engineering solutions thus typically impose parametric design fires in a performance-based environment; data on the performance of CFS columns in design fire scenarios are not currently

available. Real fires (localized or travelling) may also impose non-uniform heating which may induce column curvatures and the formation of plastic hinges or thermal curvatures leading to secondary moments in real structures. This may also be important for CFS columns forming part of a compartment wall or building façade where one-sided heating may occur.

5.2. Materials of construction

Most testing and modelling to date has focused on normal strength concrete infill whereas current practice is to use higher strengths. High strength concrete is known to be prone to spalling, to suffer proportionally greater losses in compressive strength on heating, and to display lower dilatency on loading, all of which may affect its response to loading in fire. Few tests have been performed on such columns (either constitutive, thermo-mechanical, or full-scale). Research (Kodur and Lie 1995, Kodur and Lie 1996a) has suggested that FIB infill (whether normal strength or high strength) can provide similar fire resistance as RC infill (under concentric loading) although only limited data are available and the mechanisms of the improved response are neither confirmed nor understood.

5.3. Sectional properties & response

The effects of differential thermal expansion and gap formation on the heat transfer within, and structural response of, CFS columns needs to be better understood. The size and timing of gap formation has been shown to affect heat transfer calculations for CFS columns. Column sizes being used commonly in high rise buildings are typically in excess of 600mm and can exceed 1600 mm, with plate thickness of 25 mm and more. No testing has been done (or is foreseeable) on columns of this size, meaning that a fundamental understanding of the underlying mechanics is needed to extend models and develop defensible designs. The bond-slip response between infill concrete and steel tube has received relatively little attention but clearly has relevance for load introduction or when bending is present.

5.4. Mechanical loading during fire (Full frame response)

While a few tests with eccentric loading have been reported, very little information is available for the most practically interesting cases of unprotected FIB infill and protected FIB and PC infill columns. For perimeter/edge elements in steel frames and Diagrid frames, the potential effects of bending moments on CFS columns and the formation and location of plastic hinges in the fire limit state need to be rationally assessed, particularly for unbraced structures. The appropriate effective length of CFS columns in fire has received considerable research attention (Wang 2005), yet available guidance (based almost entirely on computational modelling) may be unconservative. How, why, and where local buckling might occur and how this might affect global failure of a CFS column remains unclear (Wang 2005), and the ability to accurately predict column failure modes is marginal.

5.5. Connections and load introduction

It is critically important in buildings incorporating CFS columns to ensure that loads from beams and floor plates can be transferred into the concrete core, both during ambient design and in design for fire. Various methods to accomplish load transfer in these members are available, including internal shear

connectors or through plates. Very few studies are available on the heat transfer or structural performance of beam and floor plate connections to CFS columns during fire (Ding and Wang 2009) and additional research is needed to identify robust, convenient, and economical beam-to-CFS column connections.

6. Fire resistance design codes for CFS columns

The most common fire resistance design approaches for CFS columns are those currently suggested in North America (Kodur 2007) and Europe (CEN 2005). While other approaches are available (e.g., DL/T 1999, Guobiao (SAC) 2005, Chinese Military 2000) these are not considered here.

6.1. North American design approach

Current North American procedures for the fire resistance design of CFS columns are based on work performed at NRCC. Kodur's 2007 (Kodur 2007) state-of-the-art review notes that the National Building Code of Canada (NBCC 2005), ASCE-29 (1999), ACI 216 (2007) and AISC Fire Guide (Ruddy *et al.* 2003) all use a semi-empirical design equation developed at NRCC

$$R = f \cdot \left(\frac{(f'_c + 20)}{(KL - 1000)} \right) \cdot D^2 \cdot \sqrt{\frac{D}{C}} \quad (1)$$

Eq. (1) was developed using a computer analysis program developed at NRCC (Kodur 2007) which was validated/calibrated against tests conducted by NRCC and CIDECT (Kodur 1999). In this equation the fire resistance, R , is a function of the concrete compressive strength, f'_c , the column's effective length, KL , the diameter or width of the column, D , the applied load, C , and an empirical modification factor, f (calibrated using NRCC's sectional analysis computer program (Kodur 1999)). Table 5 provides the recommended values of this factor.

A unique feature of this approach is that it explicitly accounts for the beneficial effects of including steel fibre reinforcement within the infill concrete. Equation 1 was developed from tests/modelling that explicitly used the E119 (ASTM 2007) fire and it does not allow for other fire scenarios. This restricts its usefulness for performance-based structural fire design.

6.2. European approaches

Eurocode 4 (EC4) (CEN 2005) presents three alternative approaches for fire resistance design of CFS columns. The simplest approach is to apply prescriptive requirements that are given in Table 4.7 of EC4 (CEN 2005), which gives minimum sectional properties for a given load ratio and fire resistance. This method is highly conservative and is not discussed further here.

The second suggested approach, which is the focus of the current discussion, is a relatively simple calculation model given in Annex H of EC4 (CEN 2005). The Annex H approach represents a simplified sectional analysis technique which has two distinct steps: first, a temperature distribution over the cross-section is determined (using one of a number of applicable methods) for a given duration of fire exposure, and second, from this thermal analysis a calculation of the design axial buckling/crushing capacity of the column is made.

Several methods can be used to calculate the temperature field within the section after a given

Table 5 Possible values of the empirical parameter f in Eq. 1 (Kodur 1999)

Filling type	Plain concrete		Bar reinf. concrete						Fibre reinf. concrete	
Aggregate type	S ^a	C ^b	S ^a			C ^b			S ^a	C ^b
% of steel reinf.	N/A		< 3		≥ 3	< 3		≥ 3	< 1.77 ^c ≥ 1.77 ^c	
Cover to reinf. (mm)	N/A		< 25	≥ 25	< 25	≥ 25	< 25	≥ 25	< 25	≥ 25
f – Circular HSS	0.070	0.080	0.075	0.080	0.080	0.085	0.085	0.090	0.090	0.095
f – Square HSS	0.060	0.070	0.065	0.070	0.070	0.075	0.075	0.080	0.080	0.085

^asiliceous aggregate, ^bcarbonate aggregate, ^c% of steel fibres by mass.

duration of fire, ranging from detailed finite element analysis to a more simplified 1D heat transfer based on EN 1991-1-2: 2002 (CEN 2009) and EN 1994-1-2: 2005 (CEN 2005), in which material thermal properties can be assumed as code-specified constant values or temperature dependant values. Guidance on how to calculate the temperature profile within a section is widely available (e.g. Lennon *et al.* 2007). It is noteworthy that EC4 states that the thermal resistance between the steel wall and the concrete may be neglected, presumably because this is assumed to be a conservative omission. Once the temperature profile within the section at a given time of fire exposure has been established, the cross-section is discretized into elements in which the temperature is assumed to be uniform, and, using a simple spreadsheet analysis, relatively simple equations can be used to check that the design resistance of the column, $N_{fi,Rd}$, at the given time (and temperature profile) is greater than the design load in fire, $N_{fi,Sd}$.

The design resistance of the column, $N_{fi,Rd}$, is determined from the design axial buckling load of the column during fire. This is found by assuming that all materials experience the same strain at a given time and temperature and then calculating the strain at which the elastic critical or Euler buckling load, $N_{fi,cr}$, is equal to the plastic (crushing) resistance to compression of the cross section, $N_{fi,pl,Rd}$ (CEN 2005)

$$N_{fi,Rd} = N_{fi,cr} = N_{fi,pl,Rd} \quad (2)$$

$N_{fi,cr}$ is the summation of the elastic flexural rigidities of the steel tube (subscript a), the concrete (subscript c), and any internal steel reinforcement (subscript s)

$$N_{fi,cr} = \pi^2 [E_{a,\theta,\sigma} I_a + E_{c,\theta,\sigma} I_c + E_{s,\theta,\sigma} I_s] / l_\theta^2 \quad (3)$$

$N_{fi,pl,Rd}$ is the summation of the crushing strength contributions of the respective materials

$$N_{fi,pl,Rd} = (A_a \sigma_{a,\theta} / \gamma_{M,fi,a}) + (A_c \sigma_{c,\theta} / \gamma_{M,fi,c}) + (A_s \sigma_{s,\theta} / \gamma_{M,fi,s}) \quad (4)$$

In the above equations, l_θ is the buckling length in the fire situation, $E_{i,\theta,\sigma}$ is the tangent modulus of the stress-strain relationship for the material i at temperature θ and for a stress $\sigma_{i,\theta}$, I_i is the second moment of area the material i , and A_i is the cross-sectional area of material i . The above parameters are calculated as the summation of the contributions from all of the respective elements in the column's cross-section at a given instant of fire exposure.

Material models are provided in EC4 (CEN 2005) to account for temperature induced reductions in mechanical properties of the respective materials. Load eccentricity is accounted for in this analysis

method by replacing the design axial load in fire, $N_{fi,Sd}$, with an equivalent concentric load to the column, N_{equ} , which is increased to reflect the detrimental effects of load eccentricity on fire resistance. The maximum permissible eccentricity is restricted and the following equation is used:

$$N_{equ} = N_{fi,Sd} / (\varphi_s \cdot \varphi_d) \quad (5)$$

in which φ_s and φ_d are empirically-derived parameters to account for the steel reinforcement ratio and the load eccentricity; these are given graphically in EC4 Annex H.

By calculating the resistance to load at consecutive instants of fire exposure, a wide variety of fire scenarios can be analyzed using this approach. This makes this method appropriate for performance-based structural design for fire.

The most advanced approach permitted by EC4 (CEN 2005) is the suite of approaches termed ‘Advanced Calculation’ methods, in which detailed analyses (i.e., finite element models) of structures based on fundamental physical behaviour is permitted. The calculations in these models are complex, and these approaches are therefore not generally applicable to simple structural designs and are not considered in the current discussion. Eurocode 4 (CEN 2005) stipulates a number of requirements for such a detailed modelling approach. For instance, the thermal actions are to be as specified by EN 1991-1-2 (CEN 2009), and the thermal properties of the materials of these steel and concrete composites should be based on those given in EC4 (CEN 2005). The advanced approaches allow for non-linearity in both thermal and mechanical properties and responses. In theory, this permits a more realistic analysis, provided that the true mechanics involved are properly understood and modelled, and that appropriate assumptions and inputs (which are often unknown) are used.

7. Database of applicable furnace tests

An exhaustive database of results from standard furnace tests of CFS columns has been compiled by the authors and is presented in Table 1-4. The full database contains some 370 individual tests, however

Table 6 Restricted application of NRCC (Kodur 2007) and EC Annex H (CEN 2005) approaches

	NRCC			EC4 Annex H
	PC	RC	FIB	
Fire resistance (mins)	≤ 120	≤ 180	≤ 180	≤ 120
Axial load	≤ 1×	≤ 1.7×	≤ 1.1×	No restrictions
	Factored compressive resistance of the core according to Ref.(CSA 1994)			
Eccentric loads	Not considered			≤ 0.5×(<i>b</i> or <i>d</i>)
Concrete strength (MPa)	20-40	20-55		20-40
HSS size (mm)				No restriction
Circular (<i>d</i>)	140-410	165-410	140-410	
Square (<i>b</i>)	140-305	175-305	100-305	
% of steel reinf.	N/A	1.5-5	1.75	0-5
Concrete cover to reinf. (mm)	N/A	20-50	N/A	20-50
Effective length (m)	2-4	2-4.5		≤ 4.5

only 270 are for unprotected columns. Furthermore, the test data used for comparison against the respective design codes is somewhat different for each, since each of the approaches places different restrictions (e.g., size, infill type, concrete strength, etc.) on the applicable results. Table 6 shows the restrictions applied by the respective procedures.

The resulting NRCC (Kodur 2007) database includes 78 tests, with a reasonably good distribution between shape, load ratio, wall thickness, and concrete strength. The data are limited in terms of end restraint conditions, effective length, and concrete type. The EC4 Annex (CEN 2005) database contains 76 tests, with a good distribution section size, concrete strength, infill type, and load ratio, but a limited range of end restraint conditions, effective length, and initial load eccentricity. A full summary of the specific tests used in the analysis presented herein is presented elsewhere (Rush *et al.* 2011)

8. Implementation of the code-specified approaches

The NRCC (Kodur 2007) and EC4 Annex H (CEN 2005) methods of analysis are the two currently available methods of analysis that are, due to their relative simplicity, most likely to be applied by structural (or structural fire) engineers in performing fire resistance design of CFS columns. The remainder of this paper presents a statistical comparison (meta-analysis) of these two approaches by comparing their predictions against their respective applicable fire test databases. In the implementation of the two approaches, the test day material strengths, where quoted in the literature, were used with no material modification factors. Where no material data were presented, the specified values were assumed.

8.1. NRCC equations (Kodur 2007)

The NRCC (Kodur 2007) approach, being a simple equation incorporating tabulated semi-empirical parameters, is straightforward to implement for columns of known materials and dimensions. The approach is easy and expedient to use making it attractive to designers.

8.2. Eurocode 4 Annex H approach (CEN 2005)

The EC4 Annex H approach (CEN 2005), while still considered a ‘simplified’ method, requires considerable effort to determine the fire resistance time of a given CFS column. Temperature distributions over the column’s cross-section are required at various durations of fire exposure so that the capacity can be determined using Eqs. (2)-(6). This capacity must then be compared against the load demand in the fire limit state.

While a number of techniques exist to determine the temperature distribution in a circular or square CFS column during fire, finite element heat transfer analysis was used in the current study to determine the temperature distribution for each individual test in the database at one-minute intervals. The input parameters to the analysis, apart from the cross-sectional dimensions, were the thermal properties of steel and concrete, and the thermal insult (fire exposure). EN 1994-1-2: 2005 (CEN 2005) provides thermal properties for both structural and reinforcing steel, as well as for concrete. Two approaches are permitted for each material: one in which thermal properties are temperature dependant, and one in which constant values are assumed. In the current analysis, the simplifying case of constant values for thermal conductivity, λ , and heat capacity, c , was used. The specific heat capacity for steel and concrete

were therefore taken as $c_a = 600 \text{ J/kg}\cdot\text{K}$ and $c_c = 1000 \text{ J/kg}\cdot\text{K}$ for steel and concrete, respectively, and the thermal conductivities were taken as $\lambda_a = 45 \text{ W/m}\cdot\text{K}$ and $\lambda_c = 1.6 \text{ W/m}\cdot\text{K}$, as recommended for simple calculation models in EC4 Annex H (CEN 2005). This approach is conservative but is likely to be the path taken by most non-specialist designers. The thermal insults used in the majority of the tests were ISO 834 (ISO 1975) standard fires (or similar).

For simplicity it was assumed that the surface temperature of the unprotected CFS column would be the same as that of the fire. This simplifying assumption is common in the analysis of unprotected steel elements and is reasonably consistent with steel temperatures measured in tests (Kodur 1999). In reality the temperature of the fire will be slightly higher than the steel surface temperature. This will result in conservative fire resistance predictions as compared with a more rigorous surface heat flux analysis; this is not considered critical for the current paper.

Guidance on how to calculate the load capacity according to the EC4 Annex H approach is available in a number of publications (e.g., Lennon *et al.* 2007, Aribert *et al.* 2008). Typically, the steel tube is assumed to have uniform temperature and the concrete cross-section is divided into rings of equal thickness (Fig. 1) and constant temperature. Clearly, the more layers that are assumed the more refined the temperature predictions, however between five (Aribert *et al.* 2008) and ten (Lennon *et al.* 2007) layers have been suggested in the literature; in the current analysis seven layers was assumed. The temperature of each layer was determined by applying the average temperature across the layer taken from a rigorous heat transfer analysis using ABAQUS commercial finite element software.

After the temperature of each layer was determined, its mechanical properties were found by interpolation of Tables 3.1-3.4 of EN 1994-1-2:2005 (CEN 2005) applied within Eqs. (4)-(6) of the current paper.

9. Means of statistical comparison

A comparison of the two above approaches is made on the basis of statistical measures of conservatism, accuracy and precision. These measures are also used to investigate the possible influences of various parameters on the fire resistance of unprotected CFS columns. The mean percentage error (*MPE*) is used to give an indication of a model's conservatism (and, to a certain extent,

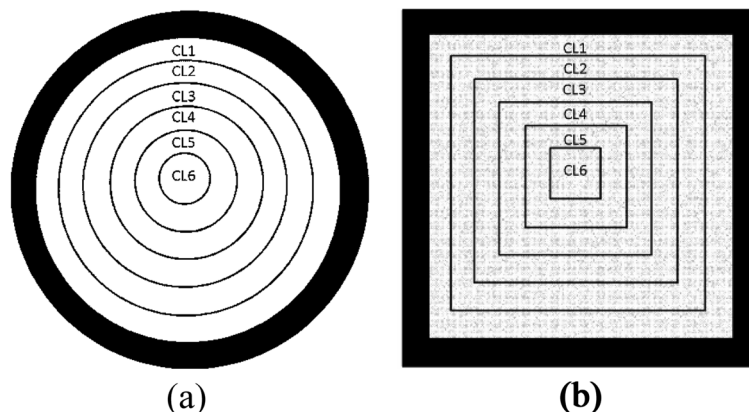


Fig. 1 (a) Circular and (b) square segments used in Annex H analysis (CL*i* = concrete layer *i*)

its accuracy of prediction) and was calculated by taking the average percent error between the model prediction and the experimentally observed value for each result in the database for each design approach. The mean absolute percentage error (*MAPE*) gives an indication of a model's accuracy, and was calculated by taking the average *absolute* percent error of prediction. Finally, the standard deviation of the error of prediction (σ) provides an indication of a model's precision. If it is assumed that the errors of prediction are normally distributed about their mean; σ can be a measure of the statistical confidence in a model's predictive ability.

10. Quantitative comparison of approaches

Fig. 2(a) shows the fire resistance predictions of the NRCC (Kodur 2007) design approach versus the observed fire resistance times for each applicable column in the database. Fig. 3(b) shows the same comparison for the EC4 Annex H (CEN 2005) approach. Both figures include a line showing one-to-one (1:1) prediction. Points that fall above the 1:1 line represent under-predictions whereas points below the line are conservative. The dashed line represents the mean error of prediction (i.e., a line through the origin that seeks to minimize the total error of prediction).

The *MPE* of prediction for the NRCC design approach is 17%, meaning that on average this approach over-predicts the fire resistance by 17%; this approach is unconservative on average. Furthermore, the *MAPE* of the NRCC (Kodur 2007) approach is 40%, meaning that the average magnitude of the error of prediction is 40%. The standard deviation of the error of prediction is 59%, meaning that the approach's mean error of prediction is 0.29 standard deviations above the 1:1 line. The statistical confidence in the model is relatively low and only about 40% of the model predictions are expected to fall on the conservative side of the 1:1 line.

Fig. 2(b) shows that the predictions of the EC4 Annex H (CEN 2005) approach are similarly imprecise, with a *MAPE* of 64%. This approach is less accurate than the NRCC (Kodur 2007) approach, with a *MPE* of -64%. However, this approach is conservative on average. The standard deviation of the error of prediction is 16% in this case, such that the mean error of prediction is 3.94

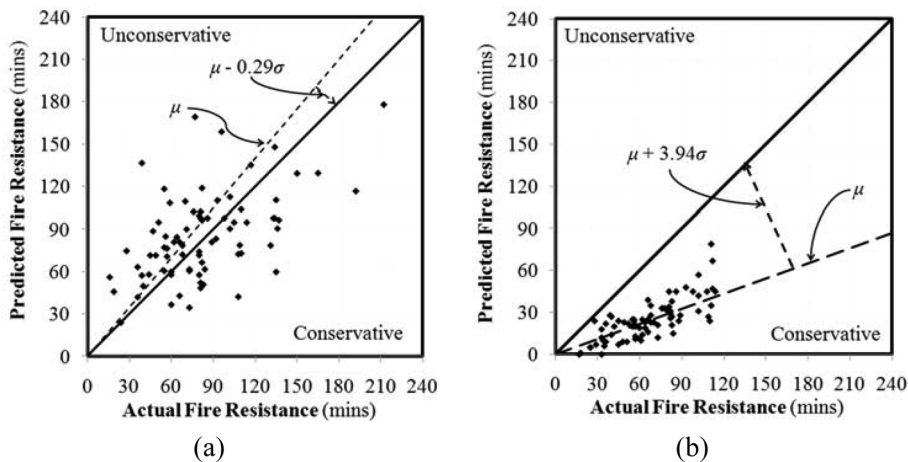


Fig. 2 Predicted fire resistance versus observed fire resistance based on (a) NRCC (Kodur 2007) design approach, (b) EC4 Annex H (CEN 2005) design approach

standard deviations below the 1:1 line; almost all predictions are statistically expected to be conservative.

Fig. 2 shows that considerable imprecision and variance exist in both design approaches, leading to a lack of statistical confidence in both of them. The lack of accuracy could be because the data provided by the testing reports omit key pieces of information (true effective length, for instance) or due to the model not properly capturing the true mechanics of CFS columns in fire or missing other key aspects of behaviour observed in tests (alternative failure modes, local buckling, etc.).

The EC4 Annex H (CEN 2005) approach considers eccentric loads if present, using modification factors applied to the loads to create an effective concentric design load. When test results were partitioned based on eccentric or concentric loading, the conservatism increased for the eccentric loading cases. The *MPEs* for concentric and eccentric loading are -60% and -77%, respectively, the *MAPEs* are 60% and 77%, respectively, and standard deviations are 0.15 and 0.13, respectively. The reasons for this are not known, although it is likely that the additional conservatism arises due to the conservative assumptions required to include eccentric load effects in a simple way.

10.1. Parameters affecting performance of the design approaches

10.1.1. Effect of column cross-sectional size

Fig. 3(a) shows the effect of column diameter on the *MPE*, *MAPE*, and σ of fire resistance prediction for the NRCC (Kodur 2007) approach. This figure shows that the NRCC (Kodur 2007) approach is generally less conservative for columns of larger diameters, and that it considerably over estimates fire resistance (on average). This is significant because columns in real buildings tend to fall at the larger end of the spectrum of possible column sizes; the result suggests that additional research on columns of large diameter is warranted if the NRCC procedure is to be applied in practical situations. The statistical confidence in prediction for columns with diameters greater than 300 mm is dramatically reduced (although this is partly because relatively few test results are available). The mean error of prediction for the columns with a diameter above 300 mm lies 0.94 standard deviations above the 1:1 line; only 19% of the predictions for section sizes greater than 300 mm are likely to be conservative on the basis

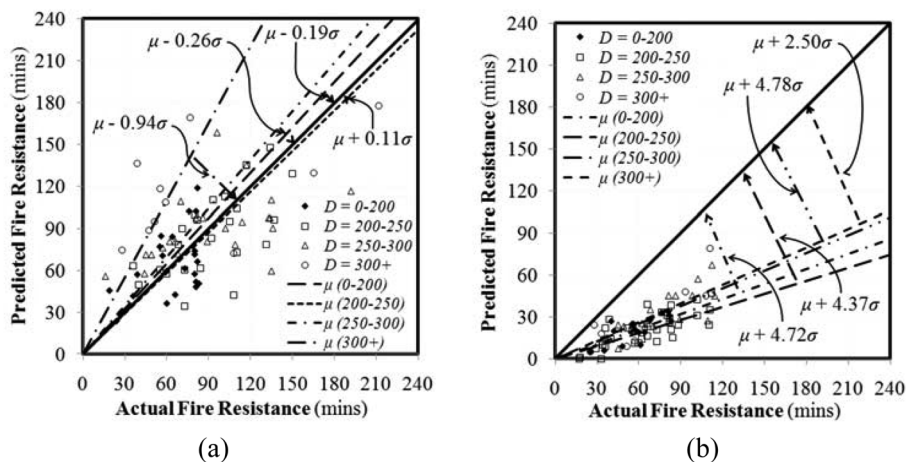


Fig. 3 Predicted fire resistance versus observed fire resistance for columns of different effective diameters based on: (a) the NRCC (Kodur 2007) design approach, (b) the EC4 Annex H (CEN 2005) approach

of the available test data. Fig. 4(b) shows that the effect of section size on the EC4 Annex H (CEN 2005) design approach is less clear, and for this approach there appear not to be any obvious differences in performance based on section size.

10.1.2. Effects of effective length and fire test load ratio

A key parameter in the calculation of predicted fire resistance times using both design approaches is the effective (global buckling) length of the column. In a real building, the effective length for design would be defined by the respective codes on the basis of the rotational restraint conditions acting at a column's extremities. This poses a problem for the comparative analyses presented herein, since some of the available test reports quote only idealised end conditions (i.e., pinned-pinned, pinned-fixed, fixed-fixed) (e.g., Kodur 1999), whereas others give both the idealized column end conditions and 'calibrated' effective lengths based on the specific testing furnace used and the judgement of the researchers (e.g., Stanke 1975). Furthermore, the available design codes give column effective lengths which are to be used for fire resistance design of columns in non-sway frames (CEN 2005, CSA 1994), and in many cases these do not match up with the effective length values quoted by researchers for furnace tests using these same end conditions. To reconcile this, the effective lengths quoted in test reports have been used when applying the respective design approaches, rather than using the design values that would be imposed by the codes (CEN 2005, CSA 1994). This is appropriate since it allows a better comparison of whether the respective approaches capture the true mechanics of CFS columns' response to fire, or whether they are pure empiricism.

Fig. 4(a) shows a comparison of the NRCC (Kodur 2007) and EC4 Annex H (CEN 2005) design approaches in terms of MPE , $MAPE$, and σ when using either the quoted furnace effective length or the design effective length as given in the respective code. In this figure the databases have also been partitioned on the basis of the fire test load ratio. It is noteworthy that the comparisons for load ratios greater than 0.6 should be treated with caution, since the number of tests is too few to be considered statistically rigorous.

It is apparent in Fig. 4(a) that the NRCC (Kodur 2007) approach is most conservative and most accurate for columns with low applied loads during fire, and it becomes less accurate, less precise, and

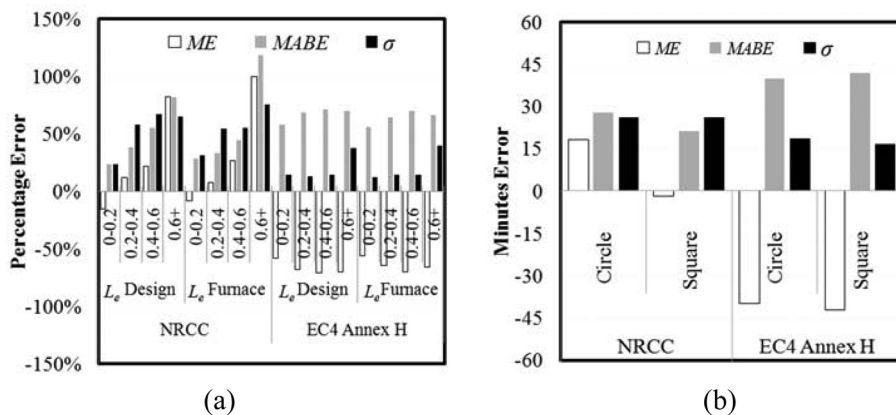


Fig. 4 Comparison of MPE , $MAPE$, and σ for NRCC (Kodur 2007) and EC4 Annex H (CEN 2005) approaches, considering (a) both quoted furnace (L_e Furnace) and design (L_e Design) effective length separated based on load ratio and (b) data separated based on cross-sectional shape

less conservative as the load ratio increases. While the reasons for this are unclear, this again suggests that this approach fails to capture the mechanics of CFS column response and/or failure. This is particularly important since load ratios for design are typically in the range of 0.4 to 0.6. Using the quoted furnace effective length rather than the design effective length (excluding the 0.6+ load ratios) has only a minor effect on model performance but slightly improves the predictions.

Fig. 4(a) shows that as the load ratio increases, the precision, accuracy and conservatism of the EC4 Annex H (CEN 2005) approach remain reasonably constant. As already noted this approach is much more conservative than the NRCC approach, but particularly so for columns with realistic load ratios. Using the quoted furnace effective length rather than the design effective length has almost no impact on the quality of fire resistance prediction due to the fire resistance of the column being determined when the buckling resistance and the resistance to crushing are equal. A change in effective length will not affect the resistance to crushing, but will have an impact on the buckling resistance. As Fig. 5(a) shows, the change in the effective length between the design and furnace effective lengths creates little change on the model's accuracy, suggesting that the columns tested are generally relatively stocky. The stiffness in fire for a CFS column is considerably higher than that of an unfilled tube of the same effective length; their fire resistance tends toward being governed by crushing rather than buckling.

10.1.3. Effect of cross-sectional shape (Circular or square)

Fig. 4(b) shows the effect of the shape of the column on the performance of the respective approaches. The NRCC (Kodur 2007) approach over-predicts the fire resistance of the circular columns (on average), whilst it slightly under-predicts the fire resistance for square columns. This approach accounts for column shape using a semi-empirical modification factor, f (given in Table 5). On the basis of the analysis presented herein, this factor appears to not properly account for column shape, since one should expect similarly conservative predictions for circles and squares. This is another indication that this approach fails to capture important aspects of CFS columns' response.

Fig. 4(b) shows that the EC4 Annex H (CEN 2005) approach, which takes shape into account by considering the area of concrete in each segment, takes shape equally well into account with squares being slightly more conservative than circles; again this model is more conservative than the NRCC (Kodur 2007) approach for both cases. It is unclear why both models should be more conservative for square columns, although this suggests that there may be fundamental differences in the way that square sections react to fire as compared with circular ones. Possible reasons for this could be the effects of confinement of the core concrete in circular columns, which would not be present to the same extent in square columns, or the non-uniform thermal profile over a rectangular cross-section (with hotter corners) as compared with the axisymmetric thermal distribution in circular columns.

10.1.4. Effect of testing laboratory

An issue of particular interest is whether the respective models are better predictors for columns tested in one testing laboratory versus another. Standard fire test furnaces globally are known to be different from one another in a host of potentially important respects; mostly with respect to the heat flux applied to the tested element resulting from different fuel sources, furnace linings, technique for measurement of gas phase temperature, control of the furnace, etc, but also in terms of the ratio of heated length to total length, loading configuration, rotational fixity, etc. It is interesting to ask whether the NRCC model, for instance, is a better predictor of test results obtained in the NRCC labs.

Fig. 5(a) shows a comparison of the MPE , $MAPE$, and σ for both the NRCC and EC4 Annex H design approaches, with the database partitioned based on the laboratory in which the respective tests

were performed. Six different testing locations are represented in the data: NRCC (Canada), CSTB (France) CTICM (France), Brunswick (Germany), Monash University (Australia), and Takenaka Technical Research Laboratory and General Building Research Corporation (Japan). The testing locations used in the comparison are different between the two design approaches due to restrictions on applicability of certain tests.

Fig. 5(a) shows that there are considerable differences in the predictive capabilities of the approaches when comparison is made between testing labs. This variation between the test centres may be due to the dates when the tests were carried out, with NRCC tests generally being more recent and therefore using more advanced equipment and techniques for measurement (in the 1980s and 1990s) than were available when the Brunswick, CTICM, and CSTB tests were performed (mostly during the early 1970s). Smaller data sets, such as for the most recent testing at Monsah University (2000s) have high variance due to small numbers of samples, and should therefore be treated with caution.

10.1.5. Effect of steel tube wall thickness

Fig. 5(b) shows a comparison of MPE , $MAPE$, and σ for both the respective design approaches, in this case with the data partitioned based on steel wall thickness. The NRCC approach does not explicitly consider steel wall thickness as a variable in determining the fire resistance of a CFS column, and its fire resistance predictions become less accurate, less conservative, less precise, and more variable as the wall thickness increases.

The standard deviation for the EC4 Annex H approach remains reasonably constant for changing wall thickness, suggesting that whilst not perfectly accounting for the wall thickness this approach is at least reasonably consistent. The predictions for thinner wall sections are slightly more conservative, possibly due to rapid heating of thin steel tubes with low thermal mass and rapid thermal expansion, which could cause the early formation of an air gap between the steel tube and concrete infill. This in turn would speed the temperature rise in the steel tube but slow the heat transfer into the concrete, allowing it to support the imposed load for longer than might be assumed by the Annex H procedures. The EC 4 Annex H approach may also underplay the ability of the concrete core to carry loads during fire, since this approach is slightly more conservative for columns with smaller wall thicknesses that rely more heavily on the core.

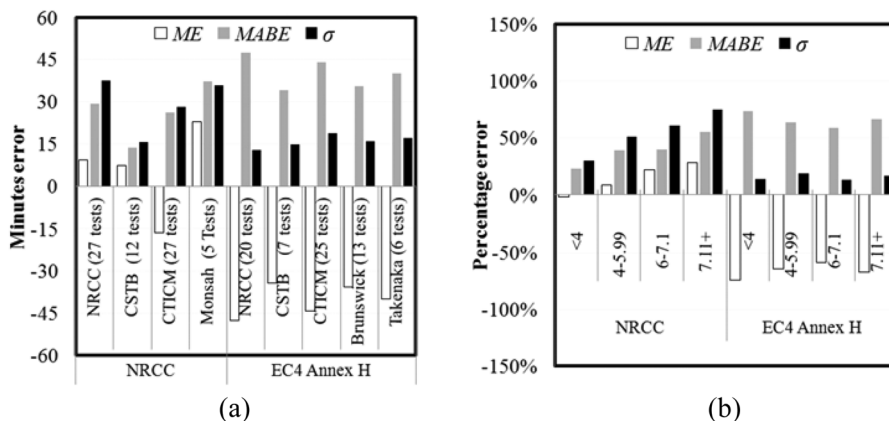


Fig. 5 Comparison of MPE , $MAPE$, and σ for both the NRCC (Kodur 2007) and EC4 Annex H (CEN 2005) design approaches with data separated based on (a) testing laboratory and (b) steel wall thickness

The above observations support the contention that both design approaches fail to capture the relevant mechanics of CFS columns during fire. The results appear to indicate that the behaviour of the steel tube is more important than assumed by either of the approaches, since their conservatism generally reduces when the steel tube plays a larger structural role. This could be due to the impacts of gap formation (as previously noted), slip between the concrete and the steel tube, or local buckling of the steel tube wall, all of which are commonly observed in fire tests of CFS columns but none of which are explicitly accounted for in either of the design approaches.

11. Conclusions and recommendations

A review of available test data, modelling approaches, and fire resistance design approaches for CFS columns (in North America and Europe) has been presented, the primary goal being to highlight gaps in knowledge and evaluate the performance of the respective approaches by comparing code predicted fire resistance times against observed fire resistance times from available standard furnace tests carried out worldwide. The focus of the comparison has been on the NRCC (Kodur 2007) and EC4 Annex H (CEN 2005) fire resistance design approaches, since these are the approaches most likely to be used in practice. The tabulated prescribed fire resistance calculation procedure given in Table 4.7 of EN 1994-1-2:2005 (CEN 2005) is highly conservative and not useful for performance-based design and has not been considered.

Both the NRCC and EC4 Annex H approaches are highly variable and provide only minimal statistical confidence in their ability to realistically predict the fire resistance of CFS columns. The EC4 Annex H approach is far more conservative than the NRCC approach, but displays similar variability; the *MAPE* is close to 40% for the NRCC approach and 65% for EC4 Annex H. In addition, because they are based on test results from standard testing furnaces, both approaches are limited in terms of their applicability to columns of realistic size, slenderness, concrete strength, concrete infill type, etc.

The NRCC (Kodur 2007) approach, while easy and rapid to use, is less accurate, less conservative, less precise, and more variable than the EC4 Annex H approach. This is because the NRCC model is a semi-empirical best fit model based both on test results and on a relatively simple numerical model (which was developed and calibrated on the basis of those same test results). As such, fire resistance is determined not by using explicit physical characteristics, but rather by applying an empirical factor which calibrated on a small number of tests. As shown when evaluated against a larger test database, the NRCC model performs poorly. This approach is only applicable to ASTM-E119 (ASTM 2007) standard fires and it is not easily applicable for performance-based design. The NRCC approach is a particularly poor predictor for large, heavily loaded, circular columns and thick-walled tubes; it fails to account for the true mechanics at play during fire.

The EC 4 Annex H approach, while also highly variable, appears to be slightly more robust than the NRCC approach. This is unsurprising, since the Annex H approach is clearly rooted in physical realities rather than being an empirical fit. This approach is highly conservative on average and shows relatively consistent variability amongst the various column parameters investigated. The effects of load ratio and wall thickness both need to be better addressed, since neither of these characteristics' influence on the observed fire resistance time appear to be accurately accounted for. This could be due to gap formation and subsequent effects on heat transfer, slip, and local buckling of the steel tube leading to failure. Importantly, this approach also allows for the use of non-standard (design) fire scenarios and is therefore applicable for performance-based design.

Additional testing is needed to develop a defensible understanding of the fundamental mechanics involved so that better design approaches can be proposed which properly account for all of the relevant parameters.

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